
भूकंप प्रतिरोध के लिए
सीमित चिनाई — रीति संहिता

**Confined Masonry for Earthquake
Resistance — Code of Practice**

ICS 91.080.30; 91.120.25

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FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

This standard is primarily concerned with the use of confined masonry for low-rise buildings for high seismic performance. The provisions of special considerations in earthquake zones relating to confined masonry building construction were briefly introduced in the National Building Code of India 2016. This standard provides detailed guidelines on design of confined masonry under different loads, such as gravity, earthquake and winds.

The standard addresses the requirements related to the design of load bearing masonry and confining element reinforcement. Also, the detailing of confined masonry walls for various level of seismic performance are enlisted.

It is assumed and expected that design of masonry work is done by qualified engineer and that execution is carried out (according to the recommendations of this standard read with other cross referred standards) under the directions of an experienced supervisor.

In the development of this standard, assistance has been derived from the following publications:

- a) EERI (2011). Seismic Design Guide for Low-Rise Confined Masonry Buildings. Earthquake Engineering Research Institute and International Association for Earthquake Engineering, USA.
- b) Eurocode 6 (2005). Design of Masonry Buildings — Part 1-1: Common Rules for Reinforced and Unreinforced Masonry Structures, EN 1996-1-1: 2005, CEN, Belgium.
- c) Eurocode 8 (2004). Design Procedures for Earthquake Resistance of Structures, Part 1-1: General Rules, Seismic Actions and Rules for Buildings. EN1998-1-1: 2004, European Committee for Standardization (CEN), Belgium.
- d) NCh2123 (2003). Albañilería Confinada — Requisitos para el diseño y cálculo (NCh 2123.Of97). Instituto Nacional de Normalización (Confined Masonry – Requirements for Structural Design), Chile (in Spanish).
- e) NTC-M (2017). Normas Técnicas Complementarias para Diseño y Construcción de Estructuras de Mampostería (Technical Norms for Design and Construction of Masonry Structures), Mexico D.F. (in Spanish and English).
- f) NT E.070 (2006). Reglamento Nacional de Edificaciones, Norma Técnica E.070 Albañilería (National Building Code, Technical Standard E.070 Masonry), Peru (in Spanish).
- g) IITK-GSDMA (2007). IITK-GSDMA Proposed Draft Provisions and Commentary on Structural Use of Unreinforced Masonry, National Information Centre for Earthquake Engineering, Indian Institute of Technology Kanpur, Kanpur, India.
- h) IITK-GSDMA (2005). IITK-GSDMA Guidelines for Structural use of Reinforced Masonry, National Information Centre for Earthquake Engineering, Indian Institute of Technology Kanpur, Kanpur, India.
- j) Brzev, S. (2008). Earthquake-Resistant Confined Masonry Construction, National Information Centre for Earthquake Engineering, Kanpur, India (www.nicee.org/confinedmasonry.php).
- k) Brzev, S. and Mitra, K. (2018). Earthquake-Resistant Confined Masonry Construction, Third Edition, National Information Centre for Earthquake Engineering, Kanpur, India.
- m) Chourasia, A. (2017). Design Guidelines for Confined Masonry Buildings, CSIR-CBRI, Roorkee, India.

Also, considerable assistance was given by the Indian Institute of Technology located at Kanpur, Gandhinagar, and Patna; and several other organizations CSIR (CBRI), Roorkee.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

CONFINED MASONRY FOR EARTHQUAKE RESISTANCE — CODE OF PRACTICE

1 SCOPE

This standard contains provisions for structural design of confined masonry, constructed with solid burnt clay bricks, fly ash bricks, solid or hollow concrete blocks, or burnt clay hollow blocks in regard to the type of materials to be used, material properties, and methods of design.

2 REFERENCES

The Indian Standards listed in Annex A contain provisions, which, through reference in the text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated in Annex A.

3 TERMINOLOGY

For the purpose of this standard, the following definitions and those given in IS 2212 and IS 1905 shall apply:

3.1 Confined Masonry Wall — A construction consisting of masonry wall and horizontal and vertical RC confining members built on all four sides of a masonry wall panel.

3.2 Confining Elements — Nominally reinforced small concrete elements built at key locations in confined masonry walls which provide deformability to masonry walls and protect them from complete disintegration even in major earthquakes.

3.3 Hollow Unit — A masonry unit with net cross-sectional area in any plane parallel to the bearing surface not less than 50 and not more than 75 percent of its gross cross-sectional area in its most unfavourable cross-section.

NOTE — Hollow masonry units should have exterior face shell thickness of not less than 15 mm. For hollow units with two to four cells, the minimum thickness of the interior webs should be 13 mm.

3.4 Horizontal Bands — A reinforced concrete runner provided in the walls to tie them together and to impart horizontal bending strength in them.

3.5 Net and Gross Area of a Masonry Unit — Net cross-sectional area of a masonry unit shall be taken as

the gross cross-sectional area minus the area of holes. Gross cross-sectional area of hollow masonry units shall be determined based on the outside dimensions of the unit.

3.6 Pier — An isolated vertical member whose horizontal dimension measured at right angles to its thickness is not less than 4 times its thickness and whose height is less than 5 times its length.

3.7 Prism — An assemblage of masonry units bonded by mortar used as a test specimen for determining masonry properties.

3.8 Solid Unit — A masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is 75 percent or more of the gross cross-sectional area in the same plane.

3.9 Shear Strength of Masonry — Shear force, v_m , resisted per unit of net cross-sectional area of masonry determined by diagonal compression test of masonry specimen (*see* Annex B) or a function of compressive strength of masonry f_m .

3.10 Specified Compressive Strength of Masonry — Compressive strength of masonry, f_m , expressed as force per unit of net cross-sectional area, required of the masonry used in construction by the contract document, and upon which the project design is based. Whenever the quantity f_m is under the radical sign, the square root of numerical value only is intended and the result has units of MPa.

3.11 Tie-beam — Horizontal confining members which resemble beams in RC frame construction and have same thickness as that of masonry wall.

3.12 Tie-column — Vertical confining members which resemble columns in RC frame construction except that they have smaller cross-section and thickness same as that of masonry wall.

3.13 Tothing — Good bonding between a masonry wall and adjacent RC tie-columns is achieved by tothing of walls by projecting bricks at the wall-to-tie-column interface.

3.14 Wall Index — The total cross-sectional area of all confined masonry walls in one direction divided by the total plan area (*see* Annex C).

4 SYMBOLS

For the purpose of this standard, the following symbols shall have the meaning as indicated below. Where other symbols are used, they are explained at the appropriate place:

- A_c – Cross-sectional area of concrete excluding reinforcing steel.
- A_m – Net area of masonry.
- A_{op} – Area of opening.
- A_s – Area of longitudinal reinforcement.
- A_{sc} – Area of transverse reinforcement.
- A_T – Area of cross-section of confined masonry wall including tie-columns.
- d – Effective depth of the wall section.
- d_b – Diameter of ties that is, both tie column and tie beam.
- E_c – Young's modulus of concrete.
- E_m – Young's modulus of masonry.
- f – Correction factor depending upon aspect ratio.
- f_a – Allowable axial compressive stress.
- f_{cc} – Permissible stress in concrete in direct compression.
- f_{ck} – Characteristic cube compressive strength of concrete.
- F_d – Design loads.
- f_d – Design strength of material.
- f_m – Compressive strength of masonry.
- f_s – Permissible steel tensile stress.
- f_t – Flexural tensile strength of masonry normal to bed joints.
- f_y – Characteristic yield strength of the reinforcing steel.
- G_m – Shear modulus of masonry.
- h – Clear wall height between floors.
- H' – Unsupported wall height between horizontal bands.
- h_c – Dimension of tie-column or tie-beam.
- h_o – Clear floor height of tie-column.
- k_s – Stress reduction factor.
- L – Actual length, unsupported length, length centre-to-centre distance of the intersecting member.
- l_w – Length of web.
- M – Factored bending moment.
- m – Modular ratio.
- M_u – Moment of resistance of confined masonry wall.
- M_{uf} – Bending moment corresponding to pure bending load condition.

- P – Factored gravity load.
- P_o – Permissible axial compressive force in confined masonry.
- P_c – Factored gravity load contributed by column.
- P_d – Design compressive axial load.
- P_m – Factored gravity load contributed by masonry wall.
- P_u – Ultimate axial compressive force in confined masonry.
- s – Tie spacing.
- S_m – Section modulus of wall section.
- t – Thickness of wall.
- V – Factored applied shear force.
- v_a – Allowable masonry shear strength.
- v_m – Masonry shear strength.
- V_u – Shear resistance of confined masonry wall.
- WI_{floor} – Wall index per floor.
- ε_c – Maximum compressive strain in concrete.
- ε_m – Maximum compressive strain in masonry.
- γ_f – Partial safety factor for load.
- γ_m – Partial safety factor for material.

5 MATERIALS

5.1 Masonry Units

5.1.1 General

Masonry units shall meet the requirements of relevant standards. Masonry units that have been previously used shall not be reused in brickwork or blockwork construction, unless they have been thoroughly cleaned and conform to the standard for similar new masonry units. Further, masonry units shall be strong enough so that they do not undergo local brittle failure.

5.1.2 Types of Units

The following types of masonry units are acceptable for confined masonry construction: solid burnt clay bricks, solid fly ash bricks, hollow concrete blocks, and solid concrete blocks. Masonry units used in construction shall comply with the following standards:

Burnt clay building bricks	IS 1077, IS 2180 or IS 2222
Concrete blocks (solid and hollow)	IS 2185 (Part 1)
Burnt clay hollow bricks	IS 3952
Other solid bricks	IS 16720

The following types of units are not permitted for confined masonry construction: masonry units with horizontal perforations, and natural stone masonry and adobe (sun-dried earthen units).

5.1.3 Compressive Strength

The minimum compressive strength of masonry units used for confined masonry buildings upto 2-storey shall be 3.5 MPa and for more than 2-storey it shall be 7.0 MPa. For concrete blocks, minimum compressive strength shall be 7.0 MPa.

NOTE — The compressive strength is determined based on the net area.

5.2 Mortar

Mortar for masonry shall comply with the requirements of IS 2250. Types M1, M2, H1 and H2 mortars as per IS 1905 shall be used for confined masonry construction.

Requirements of a good mortar for masonry structures are workability, flow, water retentivity in the plastic state and bond, extensibility, compressive strength, and durability in the hardened state. Compressive strength of mortar, in general, should not be greater than masonry unit.

Compressive strength shall not be sole-criterion for the selection of mortar. Bond strength, in general, is more important, as is good workability and water retentivity, which are required for maximum bond. Lime-based mortars should be preferred, it is desirable to sacrifice some compressive strength of the mortar in favour of improved bond.

5.3 Concrete

A minimum grade of concrete shall be M15 as per IS 456, however, higher grade of concrete can be used. The chosen concrete mix should provide adequate workability required for casting the small cross-sections of the reinforced concrete tie-columns and tie-beams. The suggested range of workability of concrete measured in accordance with IS 1199 (Part 2) is medium workability with 75-100 mm slump. The size of the coarse aggregate as per IS 383 used in construction should not be more than 12.5 mm.

5.4 Reinforcement

Steel reinforcement of grade Fe 415/500/550 (*see* IS 1786) shall be used for construction of reinforced concrete tie-columns and tie-beams.

5.5 Masonry Properties

5.5.1 Specified Compressive Strength

The specified compressive strength (f_m) for the combination of typical masonry units and mortar used for construction should preferably be determined by testing prism specimens according to the procedure outlined in Annex B of IS 1905. In the absence of test data, values for the specified compressive strength of masonry (f_m) should be obtained by multiplying permissible compressive stress (*see* 5.4.1 of IS 1905) for the combination of masonry unit strength and mortar type by a factor of 4.0 (*see* B-2.1 of IS 1905).

5.5.2 Elastic Modulus

Elastic modulus of masonry, E_m , shall be determined according to one of the procedures outlined in 5.5.2.1 and 5.5.2.2.

5.5.2.1 Prism testing

The elastic modulus shall be determined from testing of masonry prisms subjected to compression as the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism (*see* Fig. 1).

5.5.2.2 Empirical equations

In the absence of test data, the following empirical equations shall be used:

- a) For clay masonry: $E_m = 550 f_m$
- b) For concrete masonry: $E_m = 850 f_m$

5.5.3 Shear Modulus

The shear modulus, G_m shall be determined from the following empirical equation:

$$G_m = 0.2 E_m$$

6 DESIGN CONSIDERATIONS

6.1 General

Design considerations stipulated in 4 of IS 1905 shall apply along with those given under 6.2 to 6.6.

6.2 Structural Integrity

Intersecting walls shall be joined together to resist the effect of gravity and lateral loads. The walls shall be adequately bonded to elements which provide lateral support, such as floors and roof.

6.3 Building Configuration

A regular building configuration is one of the key requirements for satisfactory earthquake performance (as per IS 4326). The following recommendations related to building plan shape shall be followed:

- a) The building plan should be of a regular shape.
- b) The building's length-to-width ratio in plan shall not exceed 4 and storey height shall be less than 4 m.
- c) The walls should be built in a symmetrical manner with regards to the horizontal axes through the centre of the building plan. The walls should be placed as far apart as possible, preferably at the façade, to avoid twisting (torsion) of the building in an earthquake.
- d) There are at least two lines of walls in each orthogonal direction of the building plan, and the walls along each line extend over at least 50 percent of the building dimension in the direction of analysis at each storey level (*see* Fig. 2).

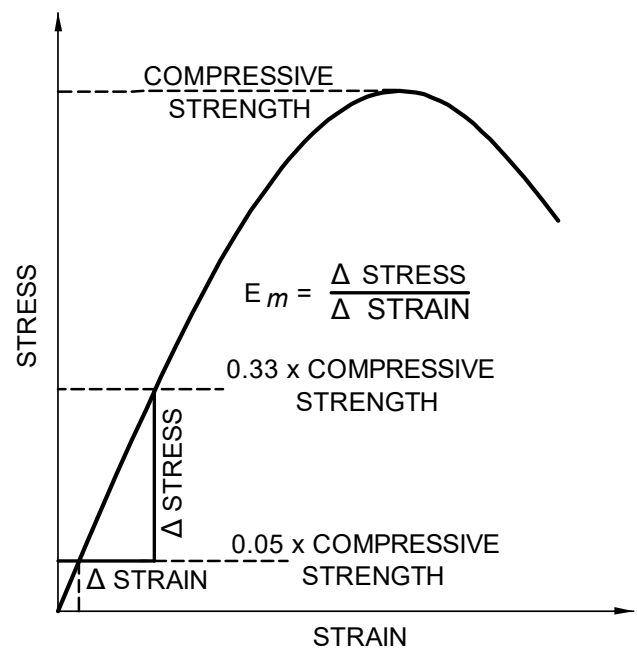


FIG. 1 CHORD MODULUS OF ELASTICITY OF MASONRY

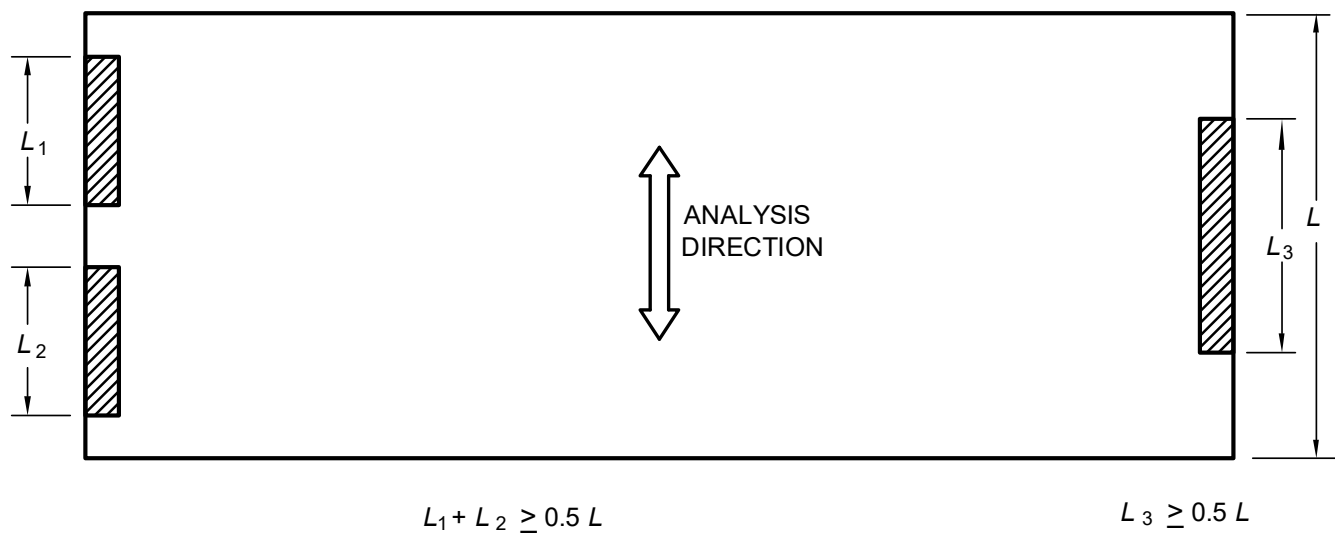


FIG. 2 ATLEAST TWO PARALLEL LINES OF WALLS ARE REQUIRED IN EACH PLAN DIRECTION

- e) The walls should always be continuous up the building height – vertical offsets are not permitted.
- f) Openings (doors and windows) should be placed in the same position on each floor.
- g) The total cross-sectional area of all walls at two adjacent floors should not be different by more than 30 percent.
- h) It is not permitted to have a moment resisting frame system at the ground floor and confined masonry wall construction at the upper floors.

6.4 Minimum Design Dimensions and Placements of Confining Elements

Requirements regarding spacing of tie-columns and tie-beams are presented in Fig. 3. Minimum tie-column and tie-beam dimensions (depth $\times$ width) shall be $150 \text{ mm} \times t$, where t is the wall thickness.

6.4.1 Placement of Reinforced Concrete Tie-Columns and Tie-Beams

Tie-columns should be provided at the following locations:

- a) wall intersections;
- b) intermediate locations in longer walls, where spacing should not exceed lesser than $1.5 h$ (h = clear wall height between floors) or 4 m; and
- c) at free ends of wall panels that provide lateral load resistance to the building (see Fig. 4).

Also, reinforced concrete tie-beams must be provided at the top of each wall and spacing between tie-beams should be preferably less than 4 m.

6.4.2 Horizontal Lintel Bands

When the unsupported height of the wall h is greater than 2.5 m, continuous horizontal lintel band shall be provided over the openings. However, continuous horizontal band is not essential for $h < 2.5$ m but lintel beam shall be provided above the openings.

6.5 Minimum Dimensions of Masonry Walls

The following dimensions shall be applicable

- a) Wall thickness (t) should not be less than 110 mm; and
- b) The maximum wall height/thickness (h/t) ratio shall not exceed 25, where h is the wall height between floors.

6.5.1 Aspect Ratio of Confined Masonry Walls

The height-to-width ratio of a wall should be kept less than 2 for the better lateral load transfer, otherwise a wall should not be considered in seismic design.

6.5.2 Parapets

RC tie-columns and tie-beams should extend to the top of the parapet, as shown in Fig. 3. When a parapet is not confined by tie-beams, height should not exceed 3 times the thickness, otherwise the height limit shall be 1.2 m.

6.5.3 Gable Walls

The top of gable should be confined with reinforced concrete tie-beams, and tie-columns located at the middle of the gable wall should be extended from the lower floor to the top of gable wall (whenever applicable), as shown in Fig. 5A. Alternatively, a gable portion of the wall can be made of timber or other light-weight material (see Fig. 5B).

6.6 Walls with Openings

6.6.1 Size of Opening

Presence of large openings may have a negative effect on seismic performance of confined masonry buildings, especially if openings are not confined. A large opening has a total area greater than 10 percent of the wall panel area, while a small opening has a total area less than or equal to 10 percent of the wall panel area.

6.6.2 Walls with Large Openings

The following three approaches shall be followed in walls with large openings:

- a) When reinforced concrete tie-columns are not provided at the ends of an opening, the panel is not considered as confined and its contribution to seismic resistance of the building should be disregarded but should be strengthened as per IS 4326, see Fig. 6A.
- b) When reinforced concrete tie-columns are provided at the opening, the confined masonry panels are considered to contribute to seismic resistance of the building but the aspect ratio H/L of these panels should be less than 2.0 as shown in Fig. 6B. Better performance can be achieved by providing both sill and lintel bands below and above the openings, respectively.
- c) If total area of openings is greater than 25 percent, both openings and masonry piers must be confined with horizontal and vertical confining elements, see Fig. 6C.

6.6.3 Walls with Small and Single Openings

The following three approaches shall be followed in walls with small openings:

- a) The opening can be ignored when it is located outside the diagonals (Fig. 7A). The entire wall cross-sectional area can be considered for seismic resistance.
- b) When an opening is located at the intersection of the panel diagonals (Fig. 7B), the panel cross-sectional area (A_T) considered for seismic resistance should exclude the opening length.
- c) When an opening is located close to one end of the panel, the panel cross-sectional area (A_T) considered in seismic resistance should use a larger pier length, as shown in (Fig. 7C).

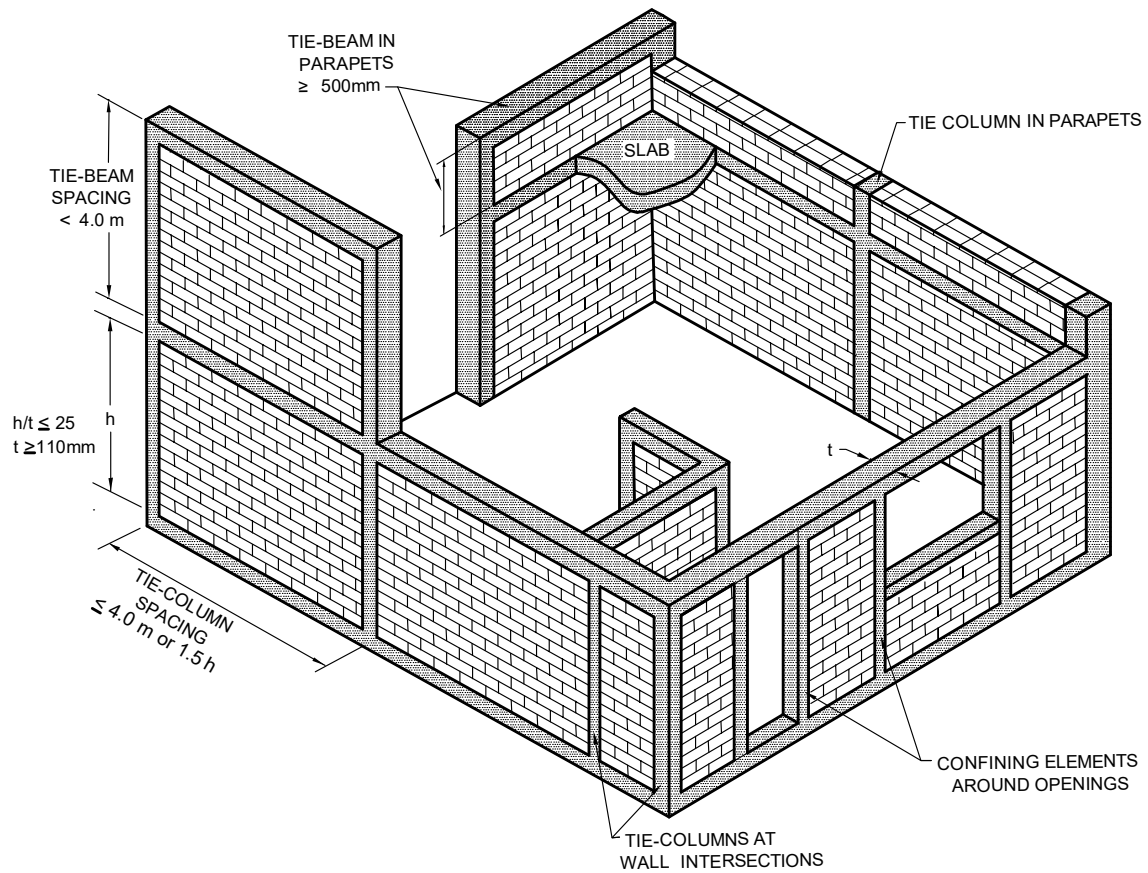


FIG. 3 REQUIREMENTS REGARDING CONFINING ELEMENTS

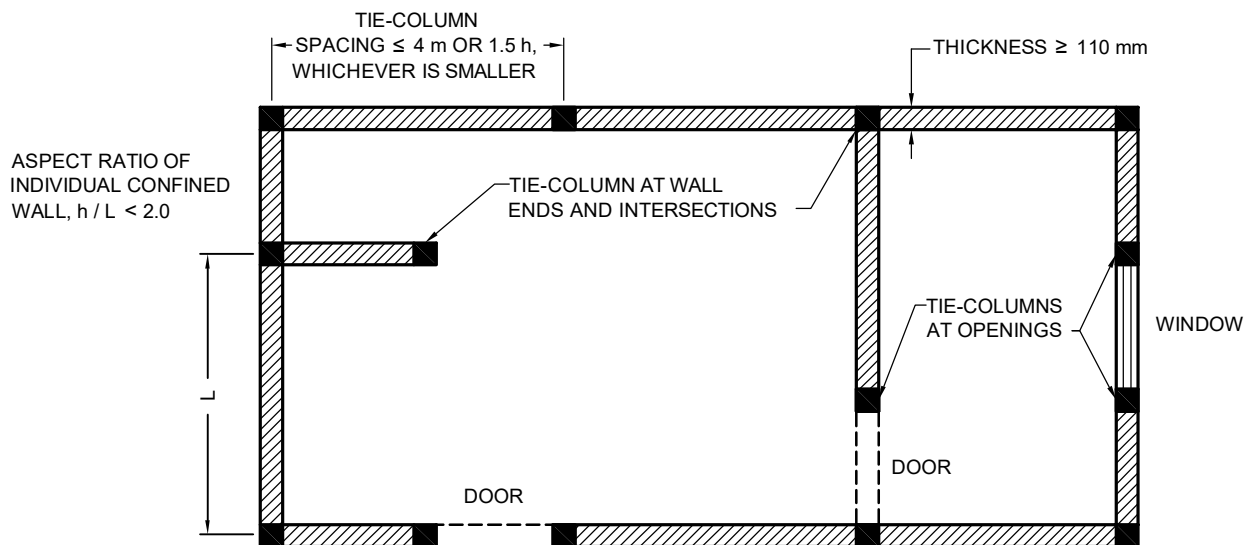
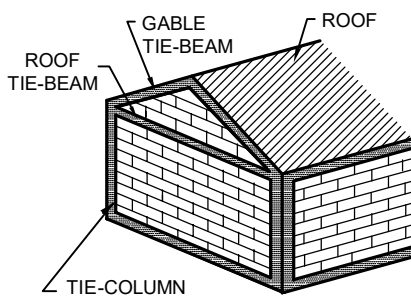
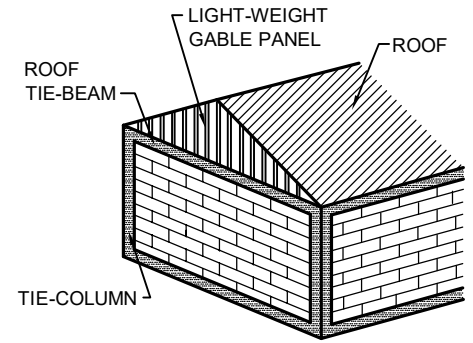
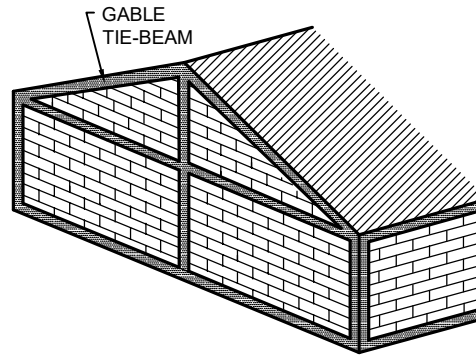


FIG. 4 TYPICAL FLOOR PLAN ILLUSTRATING THE PLACEMENT OF RC TIE-COLUMNS



5A RC Confining Elements



5B Light-Weight Gable Panels

FIG. 5 GABLE WALLS

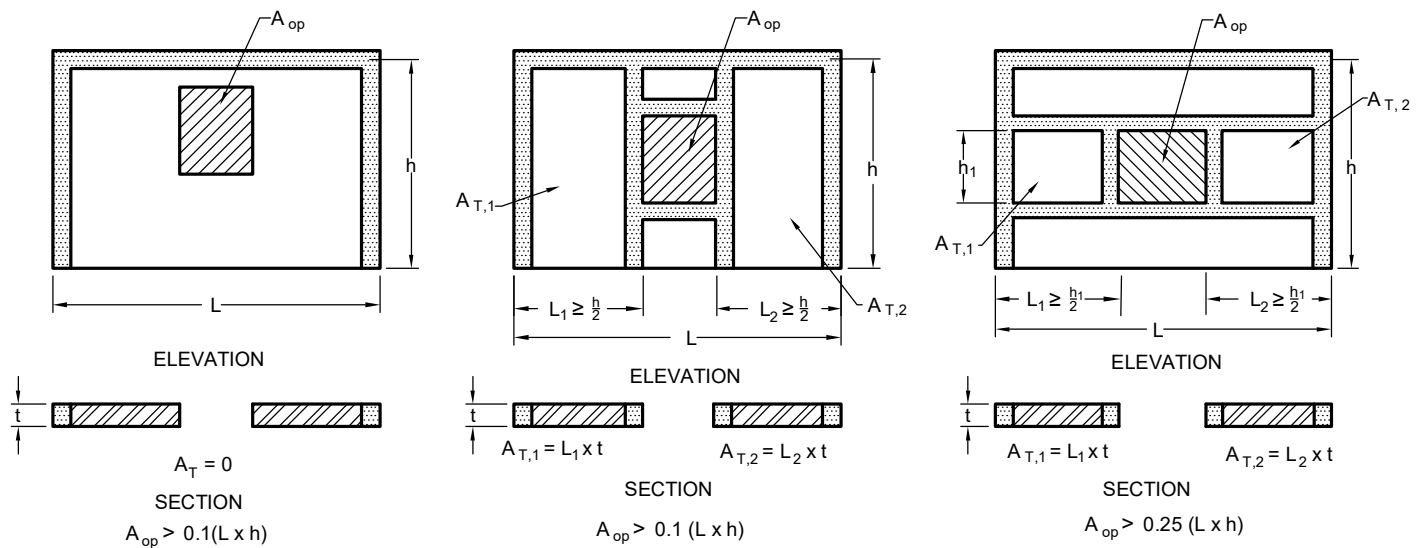


FIG. 6 MASONRY WALL WITH A LARGE OPENING

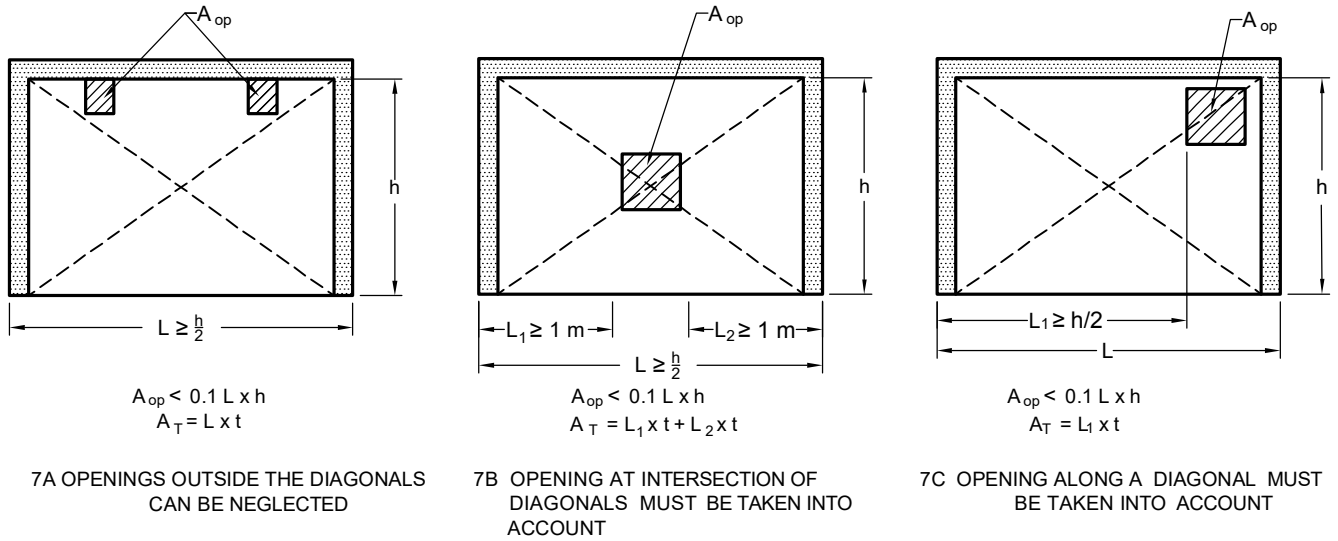


FIG. 7 CONFINED MASONRY WALL PANEL WITH A SMALL OPENING

7 STRUCTURAL DESIGN

7.1 Design Criteria

Design of structural elements should be performed according to either the working stress design method or the limit states design method.

7.2 Structural Design According to the Working Stress Design Method

7.2.1 Load Combinations

When the working stress design method is followed for the structural design of confined masonry structures, adequacy of the structure and member shall be investigated for the following load combinations:

- DL + IL
- DL + IL + (WL or EL)
- DL + WL
- 0.9 DL + EL

Permissible stresses and loads for load cases (b), (c), and (d) may be increased by one-third when wind or earthquake loads are considered in load combinations.

7.2.2 Transformed Section Properties

Confined masonry wall panel is a composite structural element which consists of a masonry wall and reinforced concrete tie-columns, and it shall be designed as a transformed section. Modular ratio (m) represents the ratio of elastic modulus for concrete and masonry. The modulus of elasticity of concrete can be assumed as per IS 456, that is, $E_c = 5000 \sqrt{f_{ck}}$ and modulus of elasticity of masonry can be taken as given in 5.5.2. Transformed section properties of a confined masonry panel used in design are illustrated in Fig. 8.

7.2.3 Permissible Compressive Force

Compressive force in confined masonry due to axial load shall not exceed that given by following equation:

$$P_o = (0.25 f_m A_m + f_{cc} A_c + 0.65 A_s f_s) k_s$$

where

A_m = net area of masonry;

A_c = cross-sectional area of concrete excluding reinforcing steel;

A_s = area of steel;

f_s = permissible steel tensile stress (Table 22 of IS 456);

f_{cc} = permissible stress in concrete in direct compression (Table 21 of IS 456); and

k_s = stress reduction factor as in Table 9 of IS 1905.

7.2.4 Combined Permissible Axial and Flexural Compressive Stress

For walls subjected to combined axial load and flexure, the compressive stress in masonry due to combined action of axial load and bending shall not exceed $1.25 f_a$ and compressive stress in masonry due to axial load only shall not exceed f_a .

7.2.5 Permissible Tensile Stress

Provisions of 5.4.2 of IS 1905 shall apply.

7.2.6 Permissible Shear Stress

The allowable shear stress for confined masonry walls shall be according to Table 1. If there is tension in any part of a section of masonry, the area under tension shall be ignored while working out shear stress on the section.

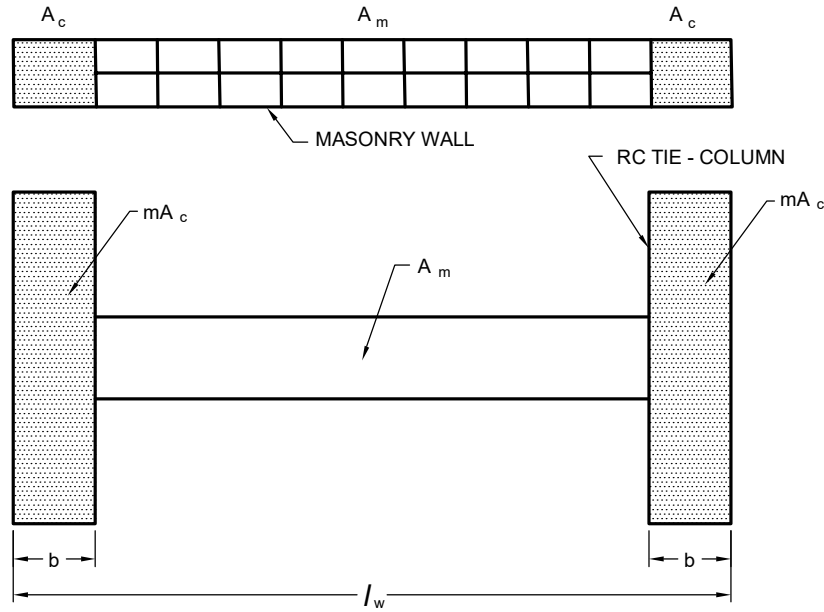


FIG. 8 TRANSFORMED SECTION PROPERTIES OF A CONFINED MASONRY PANEL

Table 1 Shear Stress for Confined Masonry, v_a
(Clause 7.2.6)

M/Vd	f_t	Maximum Allowable Shear Strength
	MPa	MPa
<1	$\frac{1}{36} \left(4 - \frac{M}{V l_w} \right)$	$\left(0.4 - 0.2 \frac{M}{V l_w} \right)$
>1	$0.083 \sqrt{f_m}$	0.4

7.3 Structural Design According to the Limit State Design Method

7.3.1 General

In the method of design based on limit state concept, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements, such as limitations on deflections and cracking. The following three limit states shall be considered in the design of confined masonry buildings: ultimate limit state (limit state of collapse), serviceability limit state, and durability limit state.

7.3.2 Ultimate Limit State

Structures and structural elements should be designed such that the design strength at any section of a structural element shall be at least equal to the design value of internal force.

Design loads (F_d) are obtained when partial safety factors (γ_f) are applied to characteristic loads (F), as follows:

$$F_d = F \gamma_f$$

The partial safety factors γ_f given in Table 2 shall be applied to various loads. The loads and load effects shall be multiplied by the relevant γ_f factors, given in Table 2, to get the design loads or design load effects for confined masonry structures.

The design strength of material (f_d) is obtained when partial safety factor (γ_m) is applied to strength of the material (f), that is:

$$f_d = \frac{f}{\gamma_m}$$

The value of partial safety factor, γ_m should be taken as 2.0 for masonry, 1.5 for concrete and 1.15 for steel.

7.3.3 Serviceability Limit State

Design according to the serviceability limit state must ensure that the structural responses (displacements, cracking and vibrations) shall be limited such that the building function is maintained and uninterrupted.

7.3.4 Durability Limit State

Confined masonry buildings shall be designed and constructed to ensure durability corresponding to a useful lifetime of the structure of 50 years. Durability shall be ensured by following provisions of applicable Indian Standards related to masonry and reinforced concrete construction.

Table 2 Values of Partial Safety Factor, γ_f for Loads
(Clause 7.3.2)

Sl No.	Load Combination	Limit State of Collapse			Limit State of Serviceability		
		DL	IL	WL or EL	DL	IL	WL or EL
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	DL + IL	1.5	1.5	–	1.0	1.0	–
ii)	DL + (WL or EL)	1.5 or 0.9*	–	1.5	1.0	–	1.0
iii)	DL + IL + (WL or EL)	1.2	1.2	1.2	1.0	0.8	0.8

NOTES

1 While considering the earthquake effects, substitute EL for WL.

2 For the limit state of serviceability, the values of γ_f given in this table are applicable for short-term effects. While assessing the long-term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

* This value is to be considered when stability against overturning or stress reversal is crucial.

7.3.5 Design Assumptions

The following assumptions shall be taken in design of confined masonry wall panels subjected to axial and/or flexural loads:

- Masonry behaves like a homogeneous material.
- The strain distribution along the wall length is based on the assumption that the wall section remains plane.
- Tensile stresses are resisted only by reinforcing steel (masonry and concrete tensile resistance is ignored).
- There is a perfect bond between steel reinforcement and adjacent concrete.
- The maximum compressive strains in masonry (ϵ_m) and concrete (ϵ_c) shall be taken equal to 0.004 0 and 0.003 respectively.
- Linear stress-strain relationship for masonry shall be considered for the design, unless more accurate relationship is determined from experimental testing of masonry prisms.

7.3.6 Stress-Strain Curves for Concrete, Masonry and Steel

The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in IS 456 (see Fig. 21 of IS 456). However, the ultimate strain in concrete should be taken as 0.003 0 in place of 0.003 5. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor, $\gamma_m = 1.5$ shall be applied in addition to this.

The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. Typical curves are given in IS 456 (see Fig. 23 of IS 456). For design purposes, the partial safety factor, $\gamma_m = 1.15$ shall be applied.

The relationship between the compressive stress-strain in masonry may be assumed to be rectangle, trapezoid, parabola or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in Fig. 9. For design purposes the partial safety factor, $\gamma_m = 2.0$ shall be applied.

7.3.7 Axial Load Resistance

Axial load resistance of a confined masonry wall (P_u) shall be determined considering the contribution of masonry and longitudinal steel reinforcement in tie-columns assuming that the steel has yielded in compression, that is:

$$P_u = k_s (0.4 f_m A_m + 0.45 f_{ck} A_c + 0.75 f_y A_s)$$

where

A_m = Net area of masonry;

A_c = cross-sectional area of concrete excluding reinforcing steel;

A_s = Area of steel;

f_{ck} = characteristic cube compressive strength of concrete;

f_y = characteristic yield strength of the reinforcing steel;

f_m = compressive strength of masonry; and

k_s = stress reduction factor as in Table 9 of IS 1905.

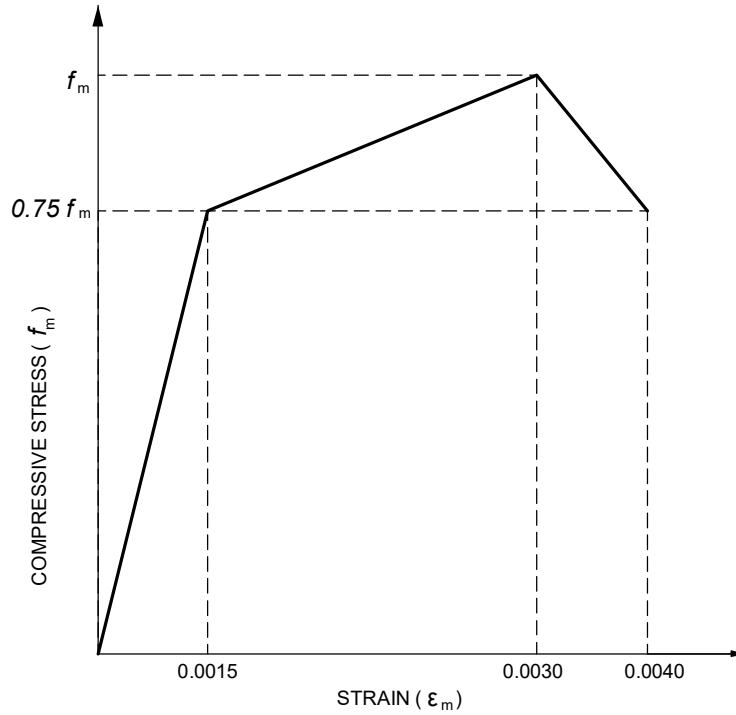


FIG. 9 ACCEPTABLE STRESS-STRAIN CURVE FOR MASONRY

7.3.8 Design of Confined Masonry Walls for Combined Axial Load and Out-of-Plane Bending

Confined masonry walls need to be designed for combined effects of axial load and out-of-plane bending (bending perpendicular to the wall surface). Bending moments may be due to eccentric gravity loads or lateral loads acting perpendicular to wall plane (wind and earthquake). Generally, confined masonry walls have sufficient resistance for earthquake force acting in out-of-plane direction, if slenderness limits in 6.5 are satisfied. For out-of-plane forces, only one-way bending may be considered that is, bending in the vertical direction as shown in Fig. 10.

The interaction of compression due to axial stresses and bending stresses should be within the following limits:

$$\frac{P}{P_u} + \frac{M}{M_u} \leq 1.0$$

Factored bending moment, M due to design factored wind pressure p_d [see IS 875 (Part 3)] acting out-of-plane over unit length b can be calculated as follows:

$$M = \frac{P_d}{8} \cdot h^2$$

where h is floor-to-floor height. The axial resistance P_u shall be calculated as in 7.3.7 by ignoring the contribution of concrete and reinforcement in

tie-column and considering A'_m . The moment of resistance M_u per unit length shall be estimated for bending in vertical direction as follows:

$$M_u = \left(\frac{f_t}{\gamma_m} + \frac{P}{A'} \right) \cdot S'_m$$

where

axial load P , should not exceed self-weight of the unit length of panel plus A'_m times 0.15 MPa;

A'_m = net area of masonry per unit length, b (mm²);

S'_m = section modulus per unit length, b (mm³); and

f_t = flexural tensile strength normal to bed joints, which may be taken as 0.30 N/mm² for Grade M1 or better mortar and 0.20 N/mm² for Grade M2 mortar.

7.3.9 Moment Resistance of Confined Masonry Walls due to Combined Axial Load and in-Plane Bending

7.3.9.1 General approach

Moment resistance (M_u) for a section of a confined masonry wall subjected to the design axial load (P) shall be determined from equilibrium of internal forces acting on the section, by following the assumptions stated in 7.3.5.

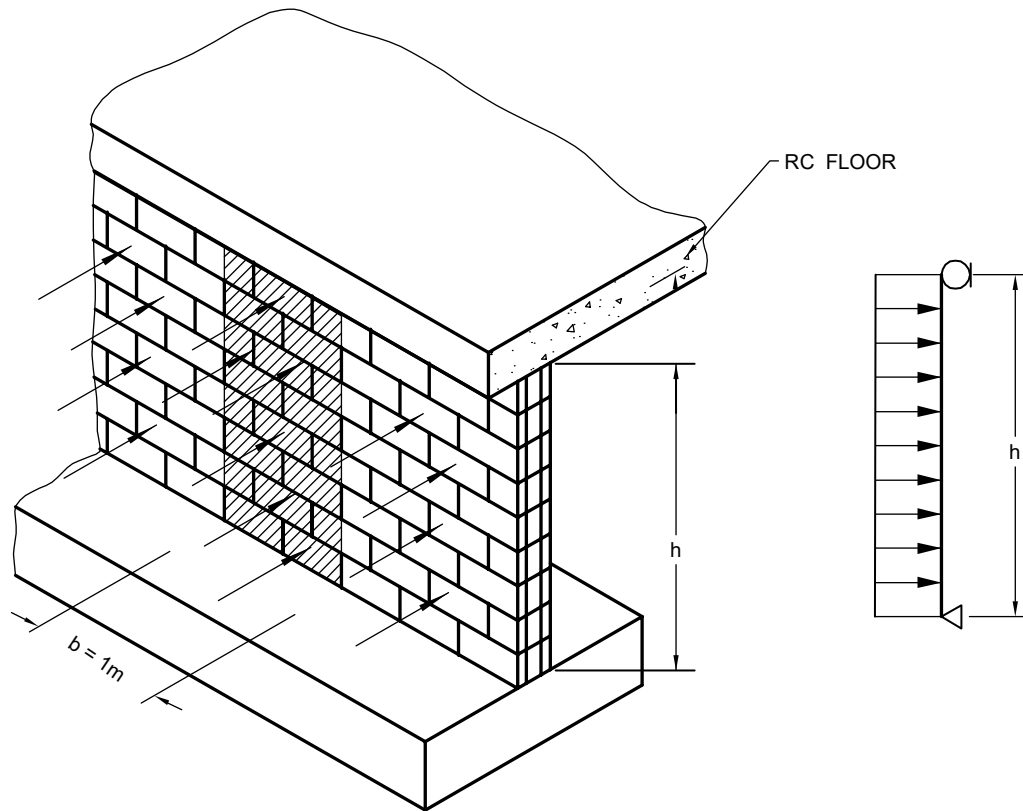


FIG. 10 UNIT LENGTH, b OF MASONRY WALL CONSIDERED FOR OUT-OF-PLANE BENDING IN THE VERTICAL DIRECTION

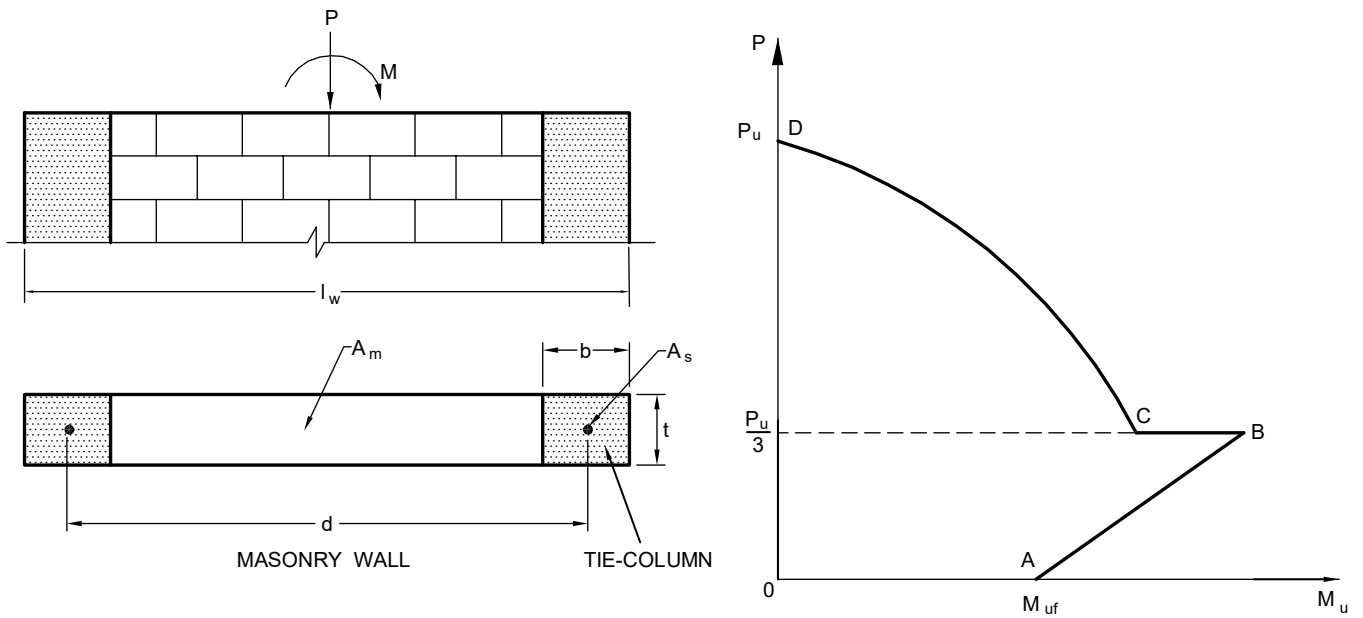


FIG. 11 SIMPLIFIED INTERACTION DIAGRAM FOR DESIGN OF CONFINED MASONRY WALLS

7.3.9.2 Simplified axial load and bending moment interaction diagram

For walls with longitudinal bars placed symmetrically in tie-columns, the following equations define a simplified axial load-bending moment interaction diagram (Fig. 11):

- a) Part A-B: when $0 \leq P < P_u/3$

$$M_u = 0.3 P d + M_{uf}$$

where, $M_{uf} = 0.87 f_y A_s (l_w - b)$ is moment resistance corresponding to pure bending load condition, and d is effective depth of the wall section.

- b) When $P = P_u/3$

Point B: $M_u = 0.1 P_u d + M_{uf}$

Point C: $M_u = \frac{2}{3} (0.05 P_u \cdot d + 1.5 M_{uf})$

- c) Part C-D: when $P > P_u/3$

$$M_u = \left[0.15 P \cdot d + 1.5 M_{uf} \right] \cdot \left(1 - \frac{P}{P_u} \right)$$

7.3.9.3 Alternate method

The P-M interaction curve for the confined masonry section can also be determined by considering the incremental values of neutral axis across the section. However, this will involve lengthy calculation by trial and error. The P-M interaction curve can also be obtained by using sectional analysis programs available for reinforced concrete members.

7.3.10 Design for Shear

7.3.10.1 General

Shear resistance of a confined masonry wall is due to combined contribution of masonry and horizontal wall reinforcement. Although reinforced concrete tie-columns contribute to shear resistance of a confined masonry wall, their contribution is not considered in the design to increase a safety margin in a confined masonry structure.

7.3.10.2 Masonry shear resistance

Shear resistance provided by masonry shall be determined from the following equation:

$$V_u = 0.8(0.5 v_m A_T + 0.4 P_d) f$$

but

$$V_u \leq 1.5 v_m A_T$$

where P_d is the design compressive axial load which shall include permanent loads only and with the partial safety factor of 1.0, A_T is the area of cross-section of confined masonry wall including tie-columns and v_m is masonry shear strength and can be assumed as follows:

$v_m = 0.16 \sqrt{f_m}$ MPa but not more than 0.6 MPa

Alternatively, v_m can be obtained from the standard test method for diagonal tension (shear) in masonry assemblages as per Annex B.

f is the corrected factor for the aspect ratio H/L of the wall and can be obtained from the following equations:

$$f = 1.55, \text{ if } H/L \leq 0.2$$

$$f = 1.7 - 0.7 \cdot H/L, \text{ if } 0.2 < H/L \leq 1.0$$

$$f = 1.0, \text{ if } H/L > 1.0$$

7.3.10.3 Alternate method (Strut-and-Tie method)

Strut-and-Tie method can be used for determining the shear capacity of confined masonry walls. This method provides a rational and consistent design approach by idealizing complex structural member with an appropriate simplified truss model. According to this procedure, based on the knowledge of direction of principal stresses, load paths are drawn through the wall in form of a truss which is analyzed for the design loads. A possible strut-and-tie model for the confined masonry wall based on its load resisting mechanism is illustrated in Fig. 12. The broken and solid lines in Fig. 12 represent the *strut* and *tie*, respectively. If the opening distribution is irregular or complex in elevation, strut and tie method can be used to estimate the in-plane resistance of the confined masonry wall.

The basic prerequisites for the strut-and-tie model are as follows:

- The equilibrium of forces at nodes should be maintained for a given set of loads.
- Tension in concrete and masonry is neglected and adequate detailing of anchorage should be provided for tie reinforcement.
- The member forces in the struts and ties are uniaxial which should not exceed the corresponding member strength at every section.
- Sufficient ductility should be available to make transition from elastic to plastic behaviour which will enable redistribution of internal forces in the members.

7.3.11 Design of Tie-Columns and Tie-Beams

7.3.11.1 Minimum amount of longitudinal reinforcement

Longitudinal reinforcement in tie-columns and tie-beams shall be proportioned to resist the corresponding vertical and horizontal components of the compression strut that develops in masonry when resisting combined gravity and lateral loads. The total area of reinforcement should be not less than 0.8 percent of the gross cross-section area of the column. For 2-storey height, the minimum area of reinforcement can be 0.6 percent.

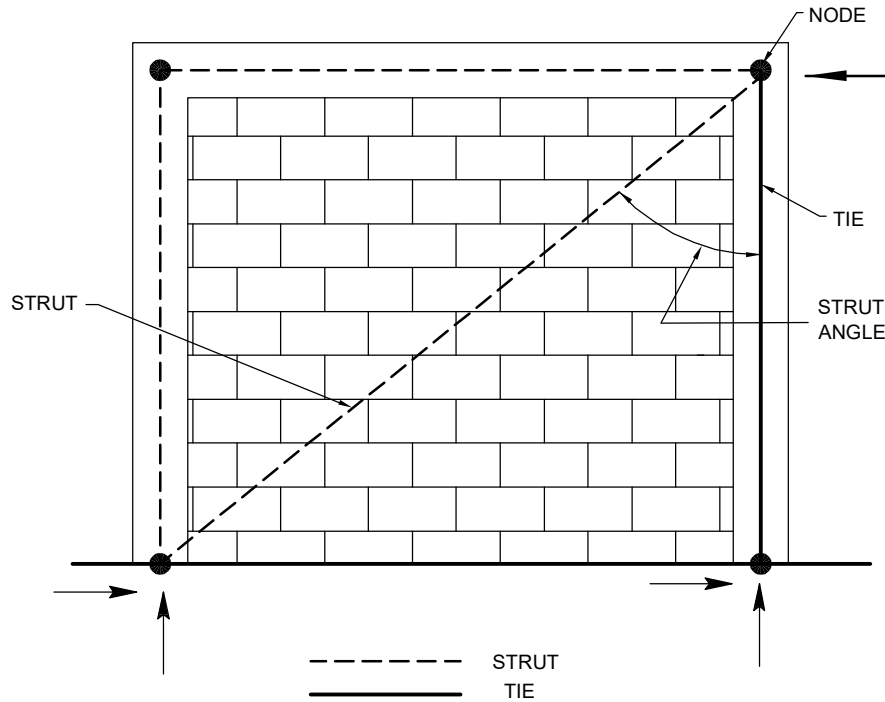


FIG. 12 STRUT-AND-TIE MODEL FOR CONFINED MASONRY WALL

7.3.11.2 Minimum amount of transverse reinforcement (ties)

Tie-columns and tie-beams shall have transverse reinforcement in the form of closed stirrups (ties) with the minimum area equal to:

$$A_{sc} = 0.0012s \times h_c$$

where A_{sc} is the total cross-sectional area, in mm², of stirrup legs effective in shear, h_c is the dimension of tie-column or tie-beam in the wall plane, in mm, and s is the tie spacing in mm.

7.3.11.3 Spacing of transverse reinforcement

Tie spacing (s) should not exceed the lesser of 200 mm and $1.5t$.

7.4 Foundations and Plinth Construction

The foundation should be constructed in the similar manner as for traditional masonry construction. Either a brick masonry footing or an RC strip footing can be used. An RC plinth band should be constructed on top of the foundation. A plinth band is essential to fully confine wall panels along their bases and prevent excessive wall damage due to building settlement in soft soil areas. Note that the longitudinal reinforcement should be extended from a RC tie-column into the plinth band, and whenever possible, into the foundation.

8 SEISMIC DESIGN

8.1 Basis

Structure should be designed for seismic forces determined as per IS 1893 (Part 1).

8.2 Methods of Analysis

8.2.1 General Considerations

For determining internal forces and moments acting on confined masonry walls, the structures may be analyzed either using the simplified method of seismic analysis, or advanced analysis methods (static and/or dynamic analysis). The simplified method can be used for buildings up to three storey high ($G + 2$), provided that the requirements stipulated in 8.2.2 have been met. The effect of openings shall be considered in lateral stiffness and strength calculations. The structure shall be modelled to adequately simulate behaviour of its critical structural elements. Elastic and shear modulus of masonry, E_m and G_m , shall be used to reflect the axial and shear stiffness expected in the structure.

Design forces and moments shall be obtained from the analysis using design loads and the corresponding load factors. A confined masonry wall shall be designed for the effects of gravity loads, and the effects of lateral loads, including shear force, in-plane bending, and out-of-plane bending moments. When the simplified

method is used, design for lateral loads may be limited to the effects of shear force.

8.2.2 Simplified Method

The simplified method is based on an idealized distribution of lateral seismic forces in regular bearing wall structures with rigid diaphragms. The method compares the shear capacity of all walls at the base of the building and the seismic base shear force determined from IS 1893 (Part 1). It is suitable for verifying seismic adequacy of regular buildings with symmetrical wall layout and predominant shear behaviour. The method does not take into account torsional effects. Due to its assumptions, application of this method is restricted to buildings with a height not exceeding three storeys ($G + 2$).

The design can be further simplified for small buildings by applying the concept of wall index according to the procedure presented in Annex C, which can be used for the design of buildings with the total ground plan area not exceeding 200 m² and the height not exceeding two storeys or 7 m.

8.2.3 Advanced Analysis Methods

Seismic analysis of confined masonry buildings four-storey or higher and buildings up to three-storey high that do not meet requirements for the application of simplified method shall be performed using dynamic or static methods according to IS 1893 (Part 1). Lateral load effects induced by earthquake shall be determined based on the relative stiffness of walls and wall segments, by considering the effect of both shear and flexure in stiffness calculations. The analysis model shall take into account stiffness of floor and roof systems, and any other restraints that may influence wall rotations.

8.3 Other Seismic Design Requirements

8.3.1 Wall Spacing

Maximum spacing of transverse walls in buildings with flexible diaphragms should not exceed 4 m. For spacing more than 4 m, the tie-beam at roof level should have a width of $L/20$.

8.3.2 Spacing of Transverse Reinforcement (Ties) in Tie-columns

For seismic zones IV and V, reduced tie spacing ($s/2$) is required at the ends of tie-columns, as shown in Fig. 13. The length over which the reduced tie spacing is used should not exceed the largest of the following values: twice the column dimension ($2h_c$ or $2t$), or $h_o/6$, where h_o is the tie-column clear floor height.

9 GENERAL REQUIREMENTS

9.1 Reinforcement Detailing

9.1.1 Concrete Cover

Minimum concrete cover to ties (transverse reinforcement) in tie-columns and tie-beams should not be less than 20 mm.

When horizontal wall reinforcement is provided, minimum clear distance between a horizontal bar and the exterior wall surface should not be less than 20 mm.

9.1.2 Bar Size and Number in Tie-columns and Tie-beams

Longitudinal reinforcement in tie-columns and tie-beams should consist of minimum 4 reinforcing bars with the minimum 10 mm diameter.

9.1.3 Ties in Tie-columns and Tie-beams

Minimum 6 mm diameter bars should be used for ties in tie-columns and tie-beams (Fig. 14). Ties should have 135° hooked ends. Hooks should be staggered, as shown in Fig. 14.

9.1.4 Anchorage of Longitudinal Bars

Longitudinal reinforcement in tie-columns and tie-beams should be anchored to develop the full specified steel yield stress. Longitudinal bars in tie-columns and tie-beams should have a 90° hooked anchorage at intersections. The lap length of the hook tails should be the largest of 20 times the bar diameter or 500 mm (Fig. 15).

Tie-column longitudinal bars at the roof level should be bent by 90 degrees and lapped with the tie-beam longitudinal reinforcement (Fig. 16).

9.1.5 Lap Splices for Longitudinal Reinforcement

Lap splices for longitudinal reinforcement should be at least 40 times the bar diameter. Longitudinal reinforcing bars should be spliced within the middle third of the column height or beam span. The splices should be staggered so that not more than 2 bars are spliced at any one location.

9.1.6 Tie-Column-to-Tie-Beam Joints

Continuity of longitudinal tie-beam reinforcement through the joint must be ensured. An example of a continuous longitudinal reinforcement is shown in Fig. 17. First tie at the ends of tie-columns (top and bottom) shall be placed as close to the joint as possible. When tie-beam depth exceeds 300 mm, vertical reinforcement in an RC tie-column must be confined by the ties, below and above the joint. An additional U-shaped stirrup must be placed at the tie-beam mid-height (Fig. 18).

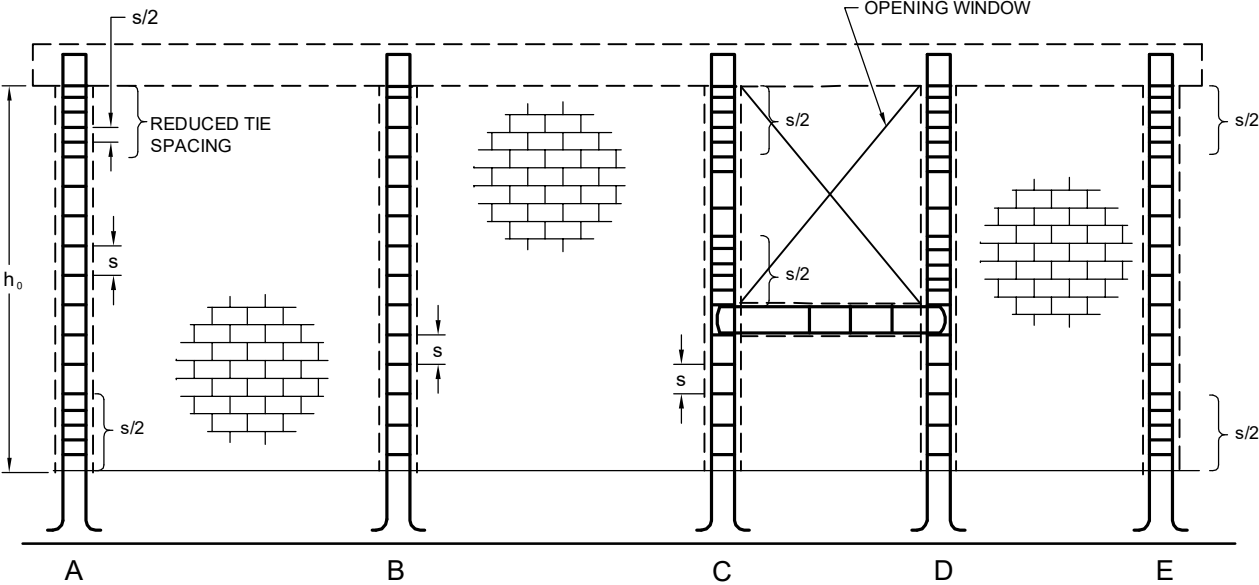


FIG. 13 TIE-COLUMN TRANSVERSE REINFORCEMENT REQUIREMENTS FOR SEISMIC ZONES IV AND V

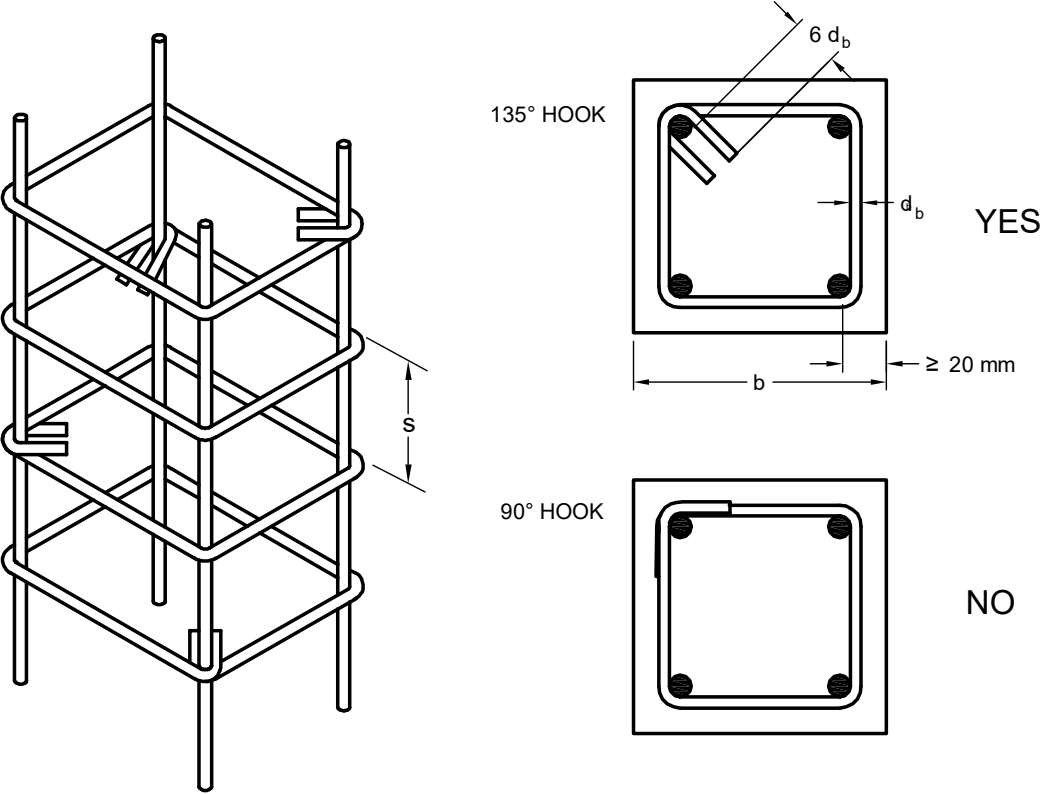
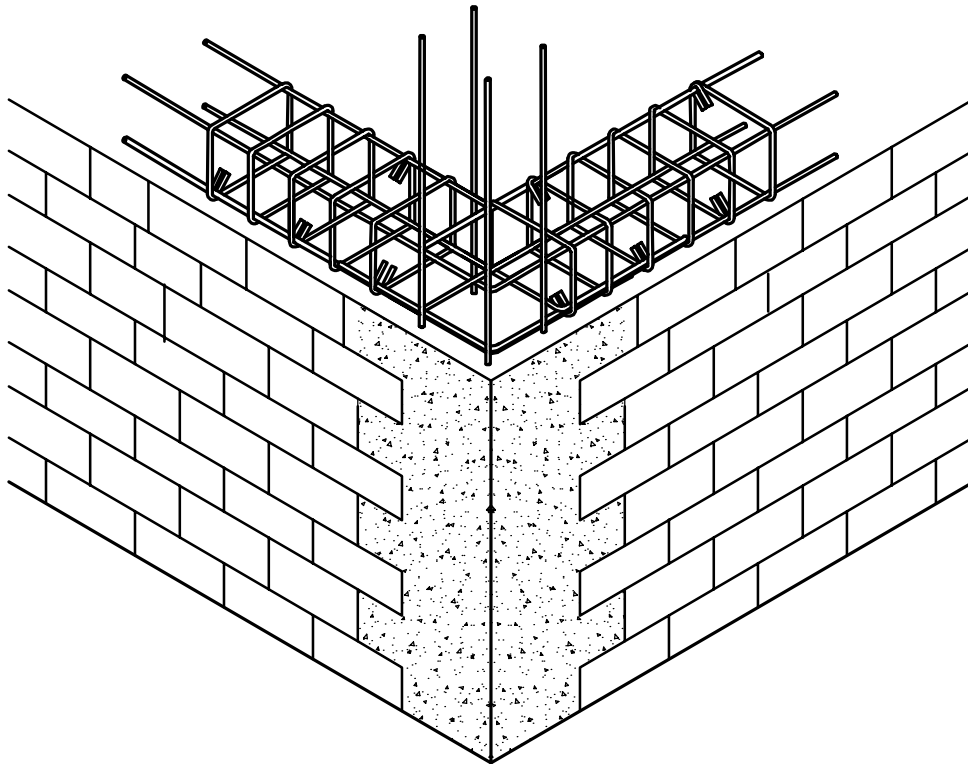
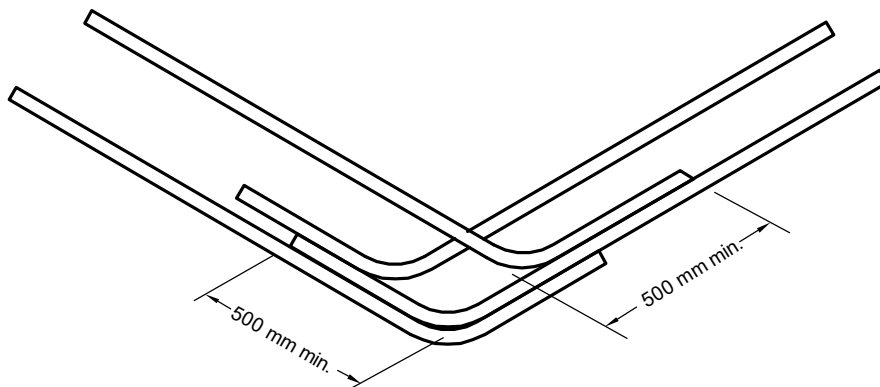


FIG. 14 LAYOUT AND DETAILING FOR TIE-COLUMNS AND TIE-BEAMS



15A WALL INTERSECTIONS



15B HOOKED ANCHORAGE FOR LONGITUDINAL REINFORCEMENT AT WALL INTERSECTIONS

FIG. 15 TIE-BEAM CONSTRUCTION

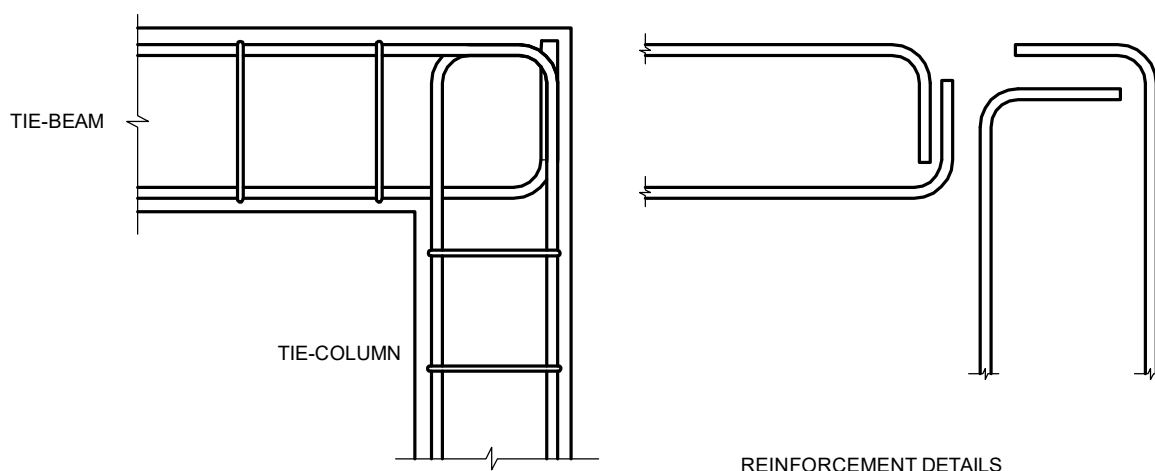


FIG. 16 ANCHORAGE OF TIE-BEAM AND TIE-COLUMN LONGITUDINAL REINFORCEMENT

Reinforced concrete tie-columns should be provided at the openings as shown in Fig. 6B and Fig. 6C. A nominal reinforcement around openings as detailed in Fig. 13 shall be provided. When reinforced concrete tie-columns are not provided at the ends of an opening then they should be strengthened as per IS 4326 or minimum two vertical bars of 10 mm diameter with 6 mm ties at 150 mm spacing should be provided.

9.2 Construction

Construction of confined masonry buildings shall satisfy the conditions specified in the applicable Indian Standards.

9.2.1 Masonry Wall Construction

9.2.1.1 General

During construction, in addition to the requirements of the sections above, the following shall be complied:

- All intersecting walls shall be connected, unless measures to assure stability and good performance are taken.
- Surfaces of construction joints shall be clean and rough. The joint shall be moistened before construction when clay units are used.
- To fulfill the requirements of excellent masonry work, Flemish bond or English bond in brickwork shall be adopted.
- During construction, necessary caution to assure wall stability at the job, shall be taken, possible horizontal pressure and loads, including wind and earthquake shall be considered.

9.2.1.2 Masonry units

The shape and dimension of masonry units, construction practices, including methods of positioning of reinforcement, placing and compacting of grout, as well

as design and detailing should be, such as to promote homogeneity of structural members, development of the bond between the grout to both reinforcement and masonry units and avoidance of corrosion of reinforcement.

9.2.1.3 Mortar joint thickness

Joint mortar shall totally cover the horizontal and vertical sides of the masonry unit. Joint thickness shall be the minimum to get a uniform layer of mortar and good alignment of the units. If industrialized units are used, the thickness of the horizontal joints shall not exceed 12 mm if horizontal reinforcement is placed in the joints, and 10 mm without horizontal reinforcement. Minimum thickness shall be 6 mm.

9.2.1.4 Toothing

The vertical interface of masonry with adjacent tie-columns shall be detailed to transfer shear forces. It shall be accepted to indent the masonry (referred to as toothing) or else, to place steel dowels or horizontal reinforcement as shown in Fig. 19.

Toothed edges should be left on each side of the wall at the interface with the tie-columns. Tothing length should be equal to one-quarter of the masonry unit length, but not less than 50 mm, as shown in Fig. 19A. When hand-made bricks are used, it is desirable to cut the brick edges, as shown in Fig. 19B. It is very important to clean the surfaces of “toothed” masonry units before the concrete has been poured.

Horizontal reinforcement anchored into RC tie-columns, also known as dowels, can be used as an alternative to toothing, as shown in Fig. 19C. Note that the dowels are not necessary when toothed edges are used. Six mm diameter bars shall be preferably used as dowels.

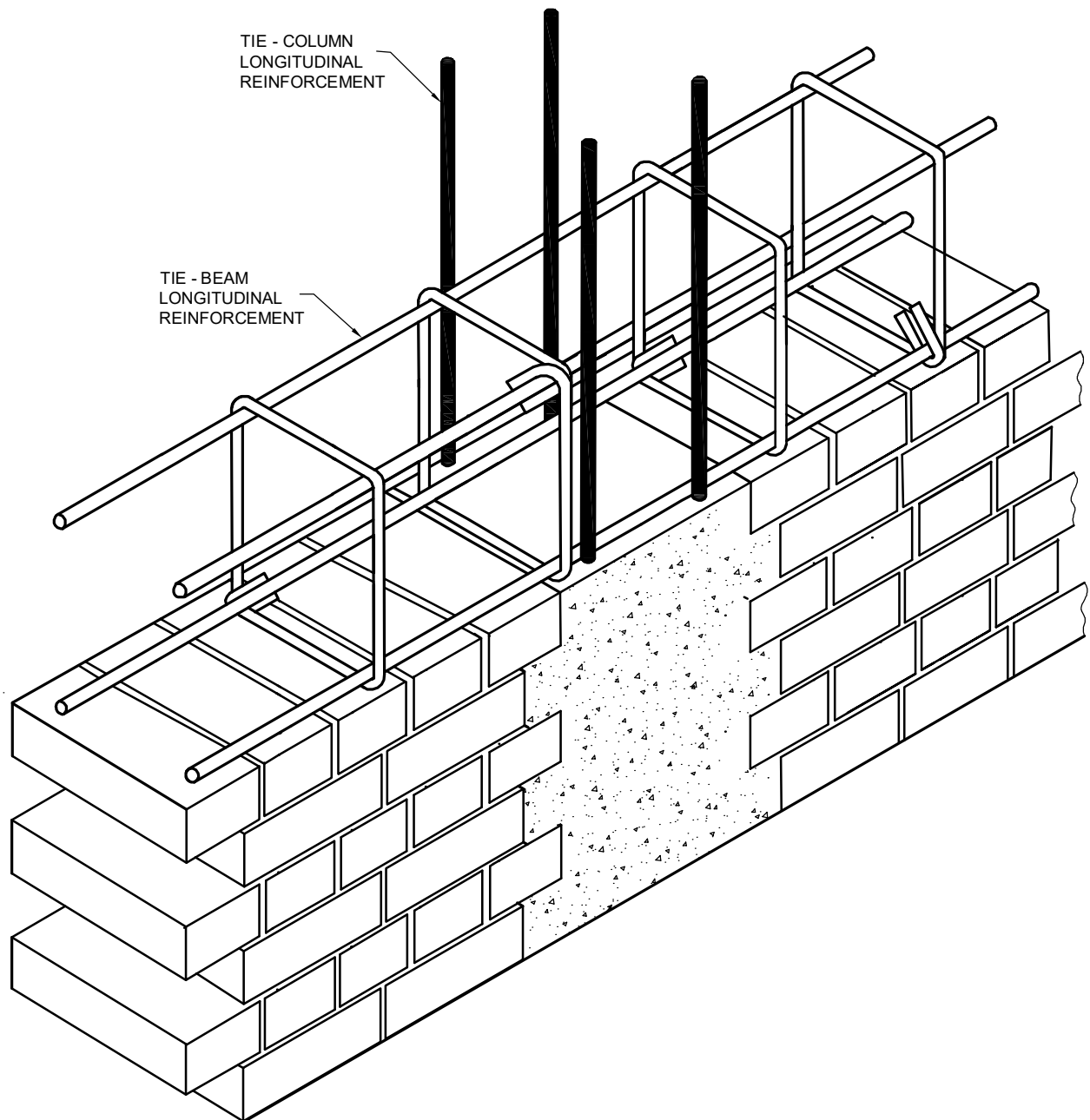


FIG. 17 CONTINUITY OF TIE-BEAM REINFORCEMENT THROUGH THE JOINT

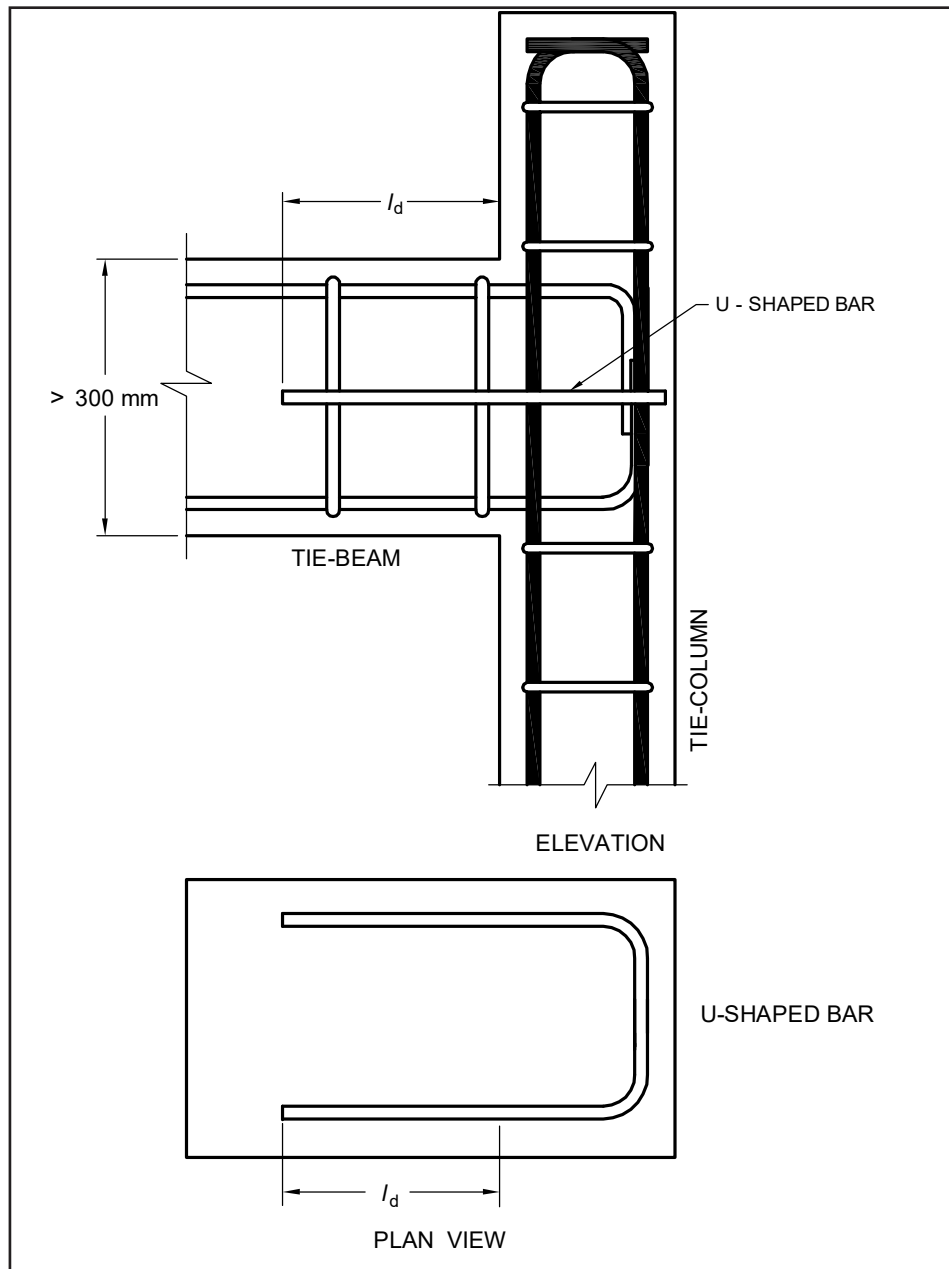
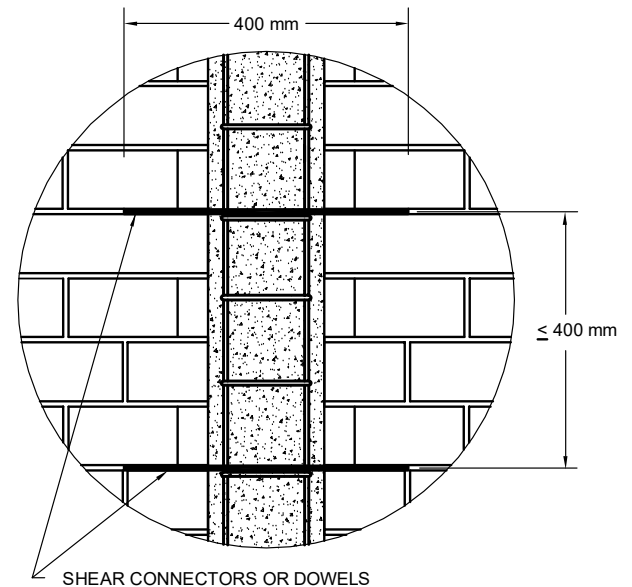
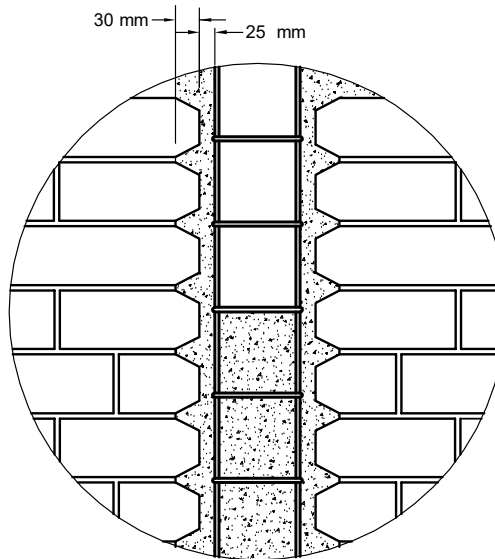
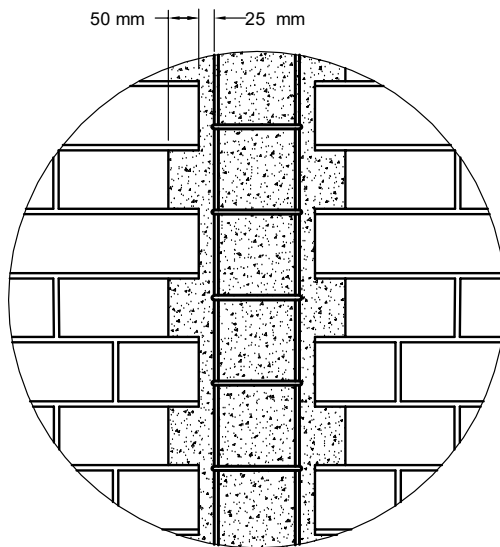
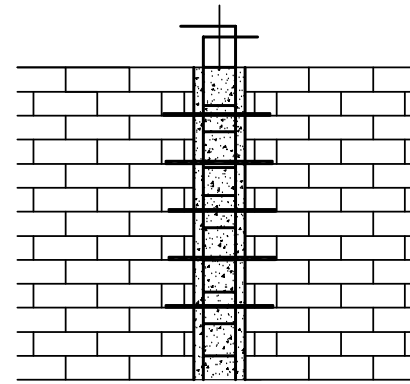
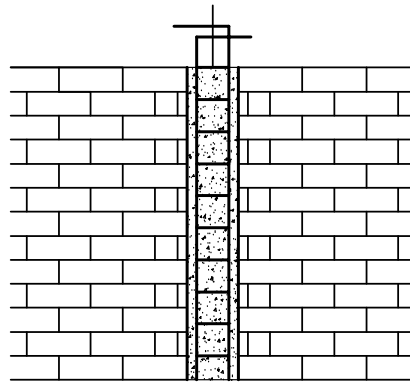
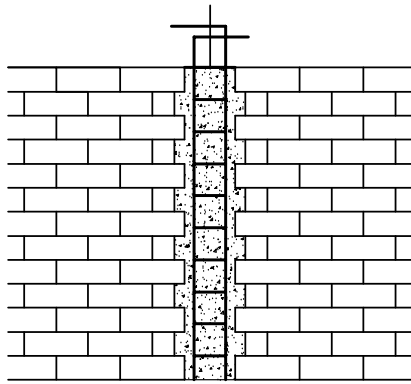


FIG. 18 ADDITIONAL CONFINEMENT FOR VERTICAL REINFORCEMENT IN THE TIE-BEAM AND TIE-COLUMN END JOINT REGION



19 A MACHINE MADE HOLLOW UNITS

19 B HAND- MADE SOLID UNITS

19 C PROVISION OF HORIZONTAL REINFORCEMENT WHEN TOOTHING IS NOT POSSIBLE

FIG. 19 TOOTHING IN CONFINED MASONRY WALLS

9.2.2 Reinforcement

Reinforcement shall be tightly secured before casting.

9.2.3 Concrete

Casting of tie-columns shall be made once the masonry wall or the corresponding part has been constructed. The masonry around the tie-columns should be saturated before the concrete has been poured.

9.2.4 Piping and Ducts

Piping and ducts shall be installed without damaging the masonry. If solid or grouted hollow units are used, grooves in the wall shall be permitted to embed piping and ducts, but the following shall need to be satisfied:

- a) Groove depth shall not exceed one fourth the thickness of the masonry wall ($t/4$);
- b) Groove shall be vertical; and
- c) Groove shall not be longer than one half the free height of the wall ($H/2$). If hollow units are used, pipes or ducts shall not be placed in cells with reinforcement. Cells with pipes and ducts shall be grouted.

It shall not be permitted to place piping and ducts in tie-columns with a structural function, whether external or internal.

ANNEX A

(Clause 2)

LIST OF REFERRED INDIAN STANDARDS

<i>IS No.</i>	<i>Title</i>	<i>IS No.</i>	<i>Title</i>
383 : 2016	Coarse and fine aggregates for concrete — Specification (<i>third revision</i>)	1905 : 1987	Code of practice for structural use of unreinforced masonry (<i>third revision</i>)
456 : 2000	Plain and reinforced concrete — Code of practice (<i>fourth revision</i>)	2180 : 1988	Specification for heavy duty burnt clay building bricks (<i>third revision</i>)
875	Code of practice for design loads (other than earthquake) for building and structures	2185	Concrete masonry units — Specification
(Part 3) : 2015	Wind loads (<i>third revision</i>)	(Part 1 : 2005)	Part 1 Hollow and solid concrete blocks (<i>third revision</i>)
1077 : 1992	Common burnt clay building bricks — Specification (<i>fifth revision</i>)	2212 : 1991	Code of practice for brick works (<i>first revision</i>)
1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement — Specification (<i>fourth revision</i>)	2222 : 1991	Specification for burnt clay perforated building bricks (<i>third revision</i>)
1199	Fresh concrete — Methods of sampling, testing and analysis: Part 2 Determination of consistency of fresh concrete (<i>first revision</i>)	2250 : 1981	Code of practice for preparation and use of masonry mortars (<i>first revision</i>)
(Part 2 : 2018)		3952 : 2013	Burnt clay hollow bricks for walls and partitions — Specification (<i>third revision</i>)
1893	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (<i>sixth revision</i>)	4326 : 2013	Earthquake resistant design and construction of buildings — Code of practice (<i>third revision</i>)
(Part 1 : 2016)		16720 : 2018	Pulverized fuel ash-cement bricks — Specification

ANNEX B

(Clauses 3.9, and 7.3.10.2)

DETERMINATION OF SHEAR STRENGTH OF MASONRY BY DIAGONAL COMPRESSION TEST

B-1 When shear strength of masonry (v_m) is to be established by tests, it shall be done in advance of the construction, using masonry specimens built of similar materials under the same conditions with the same bonding arrangement as for the structure. In making the specimens, moisture content of the units at the time of laying, the consistency of the mortar, the thickness of mortar joints and workmanship shall be the same as will be used in the structure.

The walls specimens (Fig. 20) shall have a length at least two and a half times the length of the masonry unit in one direction and adequate number of courses in perpendicular direction so that the specimen has approximately square shape. The sides of the specimen may not be less than 600 mm. The walls specimens shall be tested by loading them in compression along one diagonal. Tests shall be conducted on at least three specimens constructed with the same size and type of masonry units, mortar, and workmanship.

Specimens shall be tested after 28 days and two steel loading shoes (see Fig. 20) shall be used to apply the machine load to the specimen. For the distribution of

the applied load P_s , the ratio of the length of bearing, l_b and the width or height of specimen must be equal to or greater than 0.2. The load shall be applied at the rate of 50 to 100 kN/min and the load at failure should be recorded. Load should be applied at a uniform rate so that the maximum load is reached in not less than 1 min nor more than 2 min. Minimum thickness of steel plate used for loading shoe shall be 12 mm.

B-2 Calculate the shear stress, v_m of the specimen as follows:

$$v_m = \frac{0.707P_s}{A_{sn}}$$

where, P_s is the applied load (N) and A_{sn} is the net area of the specimen (mm²), calculated as follows:

$$A_{sn} = \left(\frac{w_s + h_s}{2} \right) \times t_s$$

where, w_s = width of specimen (mm), h_s = height of specimen (mm) and t_s = total thickness of specimen (mm).

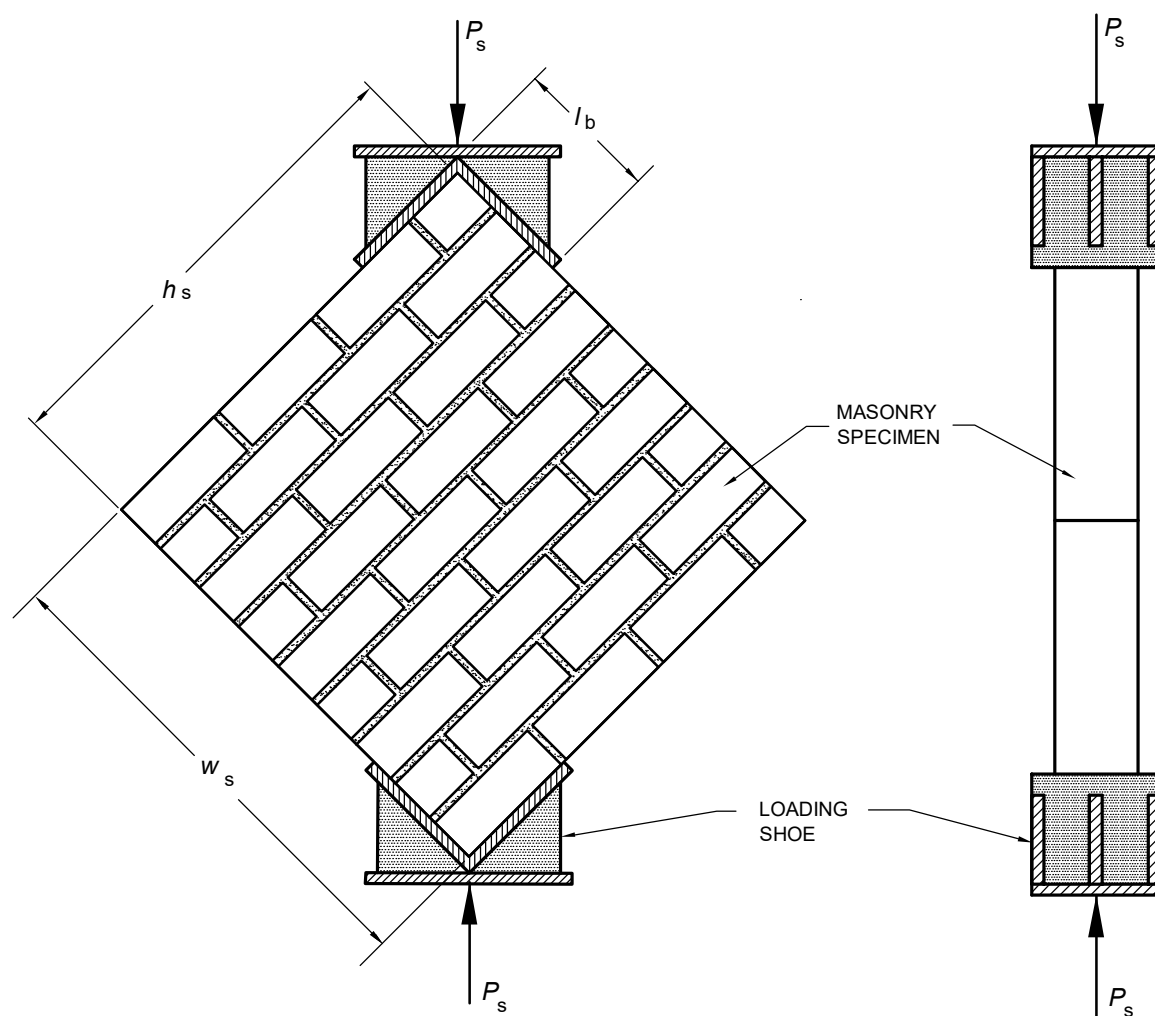


FIG. 20 WALL SPECIMEN FOR DIAGONAL COMPRESSION TEST

ANNEX C

(Clauses 3.14 and 8.2.2)

SIMPLIFIED METHOD FOR SEISMIC DESIGN OF CONFINED MASONRY BUILDINGS

C-1 Wall Index (WI), also known as wall density, is an indicator of seismic load-resisting capacity of a masonry building and can be used for seismic design of low-rise buildings with regular plan shape and elevation. The design according to the simplified method is deemed satisfactory provided that the actual wall index in each direction of the building plan is greater than or equal to the required wall index (WI_{req}) for specific building.

NOTE — The concept of wall index and illustrative examples are presented in specialist literature; Brzev and Mitra (2018).

Wall index per floor, WI_{floor} , is ratio of the sum of cross-sectional areas of all confined masonry walls in the direction considered for seismic load relative to the ground floor plan area, that is:

$$WI_{floor} = \frac{A_w}{A_p}$$

Where A_w is the cross-sectional area of all confined walls in one direction at the ground floor level. The cross-sectional area of a confined masonry wall is the product of its length (including the reinforced concrete tie-columns and the masonry wall) and thickness. A_p is the plan area of the floor diaphragm (floor slab) at the ground floor level. The area of any cutouts (openings) in the floor slab should be deducted in the A_p calculation.

Parameters for wall index calculations are illustrated in Fig. 21. The wall index shall be calculated for *each* principal direction of the building plan (X and Y). Only confined masonry walls aligned in the direction under consideration (shown shaded in Fig. 21) should be taken into account in the A_w calculations for the specific direction.

C-2 The required wall index for a building, WI_{req} , must account for the building height in terms of the number of floors, as follows:

$$WI_{req} \geq n WI_{floor}$$

Where n is the number of floor levels in a building, for example, $n = 3$ for a G + 2 building. It can be seen from the above equation that the WI_{req} increases in direct proportion with the number of floor levels in a building, and its value for a specific building depends on the following parameters:

- seismic zone of India (III, IV or V) and soil type – expressed through seismic coefficient A_h determined from IS 1893 (Part 1);

- number of floor levels in a building, n ;
- compressive strength of bricks/blocks and mortar mix; and
- average floor weight, w in kN/m², which includes the self-weight of floor slab, flooring, and walls, plus 25 percent of the imposed load.

The required WI_{floor} values for different seismic zones of India are given in Table 3. The values in Table 3 were obtained based on the ultimate limit states design method, and assuming masonry unit compressive strength of 5.0 MPa, mortar type M1 as per IS 1905, and average floor weight, $w = 12$ kN/m².

Table 3 WI_{floor} for Different Seismic Zones
(Clause C-2)

Seismic Zone	III	IV	V
WI_{floor} (percent)	1.1	1.6	2.4

Alternatively, the values of WI_{floor} can be calculated from as follows:

$$WI_{floor} \geq \frac{1.5 A_h w}{v_m / \lambda_m}$$

Where v_m is masonry shear strength as in 7.3.10.2, w is the average floor weight, A_h is the seismic coefficient, γ_m is the partial safety factor for masonry as given in 7.3.2 and W_T is the seismic weight that is used to determine the seismic base shear force V_B as per IS 1893 (Part 1), that is, $V_B = A_h \times W_T$, where A_h is determined from IS 1893 (Part 1) as follows:

$$A_h = \left(\frac{Z}{2} \right) \left(\frac{I}{R} \right) \left(\frac{S_a}{g} \right)$$

Z = the zone factor [see Table 3 of IS 1893 (Part 1)], equal to 0.16, 0.24, and 0.36 for seismic zones III, IV, and V of India respectively, I = the importance factor [see Table 8 of IS 1893 (Part 1)], R = the response reduction factor, equal to 3.0 for confined masonry ($I/R \leq 1.0$), and S_a/g = the average response acceleration [see 6.4.2 of IS 1893 (Part 1)], based on natural period of the structure, equal to 2.5 for buildings with the period of less than 0.4 s.

Different masonry properties (masonry unit compressive strength and mortar type) can be used, and the corresponding masonry shear strength, v_m , can be determined from 7.3.10.2.

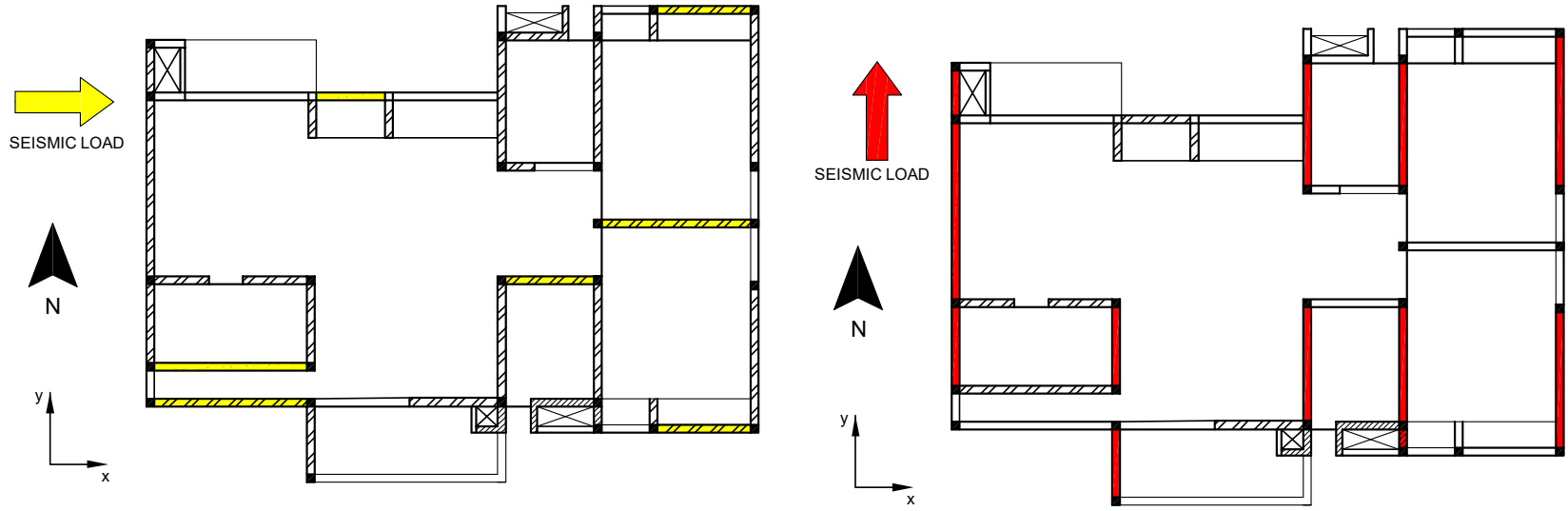


FIG. 21 PARAMETERS FOR THE WALL INDEX CALCULATIONS FOR THE PRINCIPAL DIRECTIONS OF A BUILDING PLAN (X AND Y)

ANNEX D*(Foreword)***COMMITTEE COMPOSITION**

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